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Procedia Technology 22 (2016) 4 – 12

Procedia Technology

8th International Conference Interdisciplinarity in Engineering, INTER-ENG 2014, 9-10 October 2014, Tirgu-Mures, Romania

Crack Analysis of Reinforced High Strength Concrete Elements in Simulated Aggressive Environments

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Abstract

In nowadays social and economic demands of rapid construction and durable behavior of building systems, high strength concrete is the material choice, with both rapid strength development and improved performance to aggressive environments. However, the performance of reinforced high concrete elements differs from that of ordinary concrete elements and the lack of design standards delays the implementation of the high strength concrete in private practice. Moreover, the service life performance, more specific, the durability of high strength concrete elements still remains difficult to assess, especially because of the inherent structural cracks that conduct the aggressive media to the embedded reinforcement, thus causing corrosion and possible failure of the element. The objective of the present study is to investigate the influence of the cracking pattern of reinforced high strength concrete elements exposed to accelerated corrosion on the structural behavior of the elements, with concrete cover of 25 and 50 mm. Witness specimens are used for comparison. The results of the study show that the influence of the reinforcement corrosion is more important on the 25 mm concrete covers, with the development of larger crack widths in lower stages of loading than for elements with 50 mm covers. However, the overall performance of the high strength concrete elements exposed to accelerated corrosion was above the theoretical, standard based, design capacity of structural members.

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Keywords: high strength; durability; corrosion; crack width; vertical deformation.

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Peer-review under responsibility of the “Petru Maior” University of Tirgu Mures, Faculty of Engineering
doi:10.1016/j.protcy.2016.01.002

1. Introduction

The durability of reinforced concrete structures is a subject of major worldwide concern due to the high costs of maintenance and repair when the structures are exposed to aggressive environments with two major deteriorations: the loss of element cross-section and the corrosion of the embedded reinforcement. Both lead to the reduction of the structural loading capacity, accelerated fatigue and increased deformations in service life. High strength concrete is an emerging, innovative, high performance construction material with improved durability resistance, which unfortunately is not yet fully described in design standards and thus, scarcely used in private practice. The current paper studies the performance of structural, reinforced high strength concrete elements exposed to artificial accelerated corrosion and compared to non-exposed specimens and the influence of the experimental crack width.

Nomenclature

b	width of beam cross-section (mm)
h	height of beam cross-section (mm)
c	concrete cover of the reinforcement (mm)
f_{cm}	mean value of experimental concrete cylinder compressive strength (MPa)
$f_{ct,fl}$	mean value of experimental concrete flexural tensile strength (MPa)
$f_{ct,sp}$	mean value of experimental concrete flexural tensile strength (MPa)
E_{cm}	secant modulus of elasticity of the experimental concrete (MPa)
f_{yk}	characteristic yield strength of reinforcement (MPa)
E_s	modulus of elasticity of reinforcement (MPa)
l_{eff}	effective span of the experimental beams (mm)
M	experimental bending moment at a certain loading stage (kNm)
M_{Rd}	design resistant bending moment (kNm)
M_u	ultimate, experimental bending moment (kNm)
$W_{k,EC2}$	characteristic design crack width (mm)
$W_{max,exp}$	maximum value of experimental crack width at a certain loading stage (mm)
Δ	experimental vertical deformation measured at midspan (mm)
EC2	Eurocode 2

1.1. Theoretical background

Reinforced concrete structures present structural cracks in their service life, which basically constitute an opened path for the environmental aggressive media to reach the reinforcement and initiate corrosion, with serious structural consequences. In any case, for either structural or corrosion generated cracks, researchers tried to find a solution to hinder the effects of the environment and to prolong service life. As an emerging, improved, new material, high strength concrete was also tested in reinforced elements corrosion tests, with several parameters studied and compared to normal concrete. For example, it was found that the quality of the concrete reduces the corrosion rate [1], especially if admixtures such as silica fume are used as 10% cement replacement [2]. The corrosion products were restricted to the existing cracks, filling them up and stopping further progress of the corrosion. Moreover, the asset the high strength concrete compared to normal concrete is its increased resistivity and not its higher density which could delay the propagation of the corrosion products [3]. The initiation and the propagation of the corrosion are also influenced by the crack width: less than 1 mm is reported not to have a significant impact on the corrosion process; the wider the cracks, the faster the corrosion and the moment of the failure [1,4,5,6]. However, general limitation of current design standards regarding the crack width (maximum 0.3 mm in mild environments) should be wisely used, as they should be assessed as a function of the concrete cover and the quality of the concrete [1].

Another parameter influencing the corrosion rate is the concrete cover. The larger the depth of concrete cover and the lower the water/cement ratio of the surrounding concrete [7,8], the lower the corrosion effects. The expansion of the corrosion products causes tensile stresses in the surrounding concrete and possible longitudinal cracks and loss of bond between the reinforcement and concrete and could be the estimative factors of the service life [9]. In some authors' opinion [5], the tensile strength of the concrete is the most important parameter for the formation of cracks inside the concrete cover and the subsequent spalling of the concrete and its low ductility presents an increased risk for crack formation [10].

Furthermore, the intensity of the current is related to section loss of the reinforcement. A moderate corrosion rises up to 0.5 μA/cm^2 which can produce a loss of section of 5.7 μm/year and an oxide layering up to 17.3 μm/year, whereas a stronger corrosion is produced by an intensity current of 1 μA/cm^2, which corresponds to a maximum of 11.5 μm/year and an oxide layering of maximum 34 μm/year [11,12].

The deformability of the elements is not significantly influenced by the short-term corrosion, but the presence and the distribution of the cracks can facilitate the initiation and progression of the reinforcement [13].

1.2. Purpose of the paper

The current paper studies the performance of structural, reinforced high strength concrete elements exposed to artificial accelerated corrosion and compared to non-exposed specimens in terms of crack width and deformations, with the aim of demonstrating the suitability of high strength concrete in aggressive environments, even when the corrosion process is initiated in the embedded reinforcement.

2. Materials and methods

2.1. Material properties

2.1.1. Concrete

The concrete class is C80/95, tested on cubes of 150 mm at the age of 28 days, with a mean compressive strength f_{cm} = 93.94 MPa. Furthermore, several other characteristics were tested: the mean flexural tensile strength $f_{ct,fl}$ = 9.596 MPa, the mean splitting tensile strength $f_{ct,sp}$ = 6.951 MPa and the secant elasticity modulus E_{cm} = 46905 MPa. The water/binder ratio is 0.245. The concrete composition is detailed in Table 1.

Table 1. Concrete composition - C80/95.

Components	Quantity
Portland Cement CEM I 52.5R	520 [kg/m3]
Silica fume	52 [kg/m3]
River sand (0-4mm)	530 [kg/m3]
Crushed gravel (4-8mm)	530 [kg/m3]
Crushed gravel (8-16mm)	706 [kg/m3]
Superplasticizer Glenium ACE 30	11.44 [l/m3]
Water	140 [l/m3]

2.1.2. Reinforcement

Two types of reinforcement were used for the experimental beams: European Bst500S for the tensile reinforcement with a characteristic yield strength f_{yk}=500 MPa and Romanian OB37 for the compressed bars and for the transversal reinforcement, with f_{yk}=255 MPa.

2.2. Methods

2.2.1. Description of the experimental elements and program

The experimental program and the description of the specimens are detailed in Table 2.

Table 2. Description of the specimens.

Element ID	Cross-section dimensions bxh [mm]	Concrete cover c [mm]	Type of the elements	Description of the set-up test
GDA 1-1	125x125	25		
GDA 1-2	125x125	25	a	Non exposed elements, tested to failure
GDA 2-2	125x150	50		
GDS 1-1	125x125	25		
GDS 1-2	125x125	25	b	Pre-cracked to a service limit state, exposed to accelerated corrosion, and then tested to failure
GDS 2-2	125x150	50		

Two types of beams were developed and tested: type a-exposed to accelerated corrosion and type b–non-exposed, which serve as witness specimens, as described in Table 2. The beams are different in concrete cover and cross-section, but with equal theoretical bending capacity. In order to simulate the service life of a reinforced concrete element, before subjected to accelerated corrosion, type b elements were cracked to a crack width of 0.1 mm.

2.2.2. Loading set-up

The specimens used in this study are one point loading, simply supported reinforced concrete beams (Fig. 1).

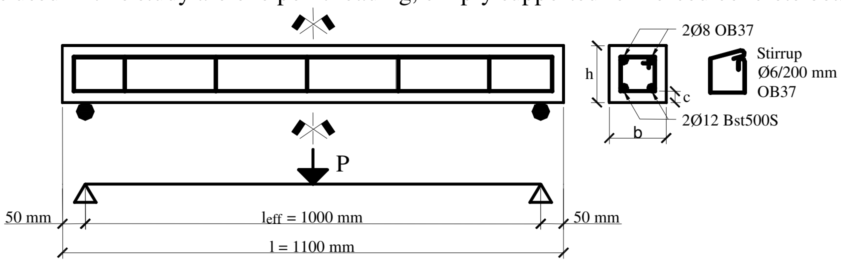

Fig. 1. Reinforcement detailing and set-up test.

2.2.3. Simulation of the aggressive environments

The corrosion of the embedded reinforcement usually takes place along several years especially if the concrete is strong and durable. Therefore, for the purpose of investigating the residual strength of reinforced concrete elements in service life, an aggressive, short-term, accelerated current was applied to the reinforcement, which would lead to loss of steel cross-section by corrosion. The set-up for the accelerated corrosion is similar to other research [14]. A schematic view of the set-up and a picture of the corrosion equipment are presented in Fig. 2.a and 2.b. The circuit of the electric current generated by an external source consisted in anode (the embedded reinforcement), the cathode (a stainless steel plate covering 3 faces of the element) and the electrolyte, a 5% NaCl solution. The intensity of the current was set to 3 mA/cm^2 and the elements were exposed for 7 days.

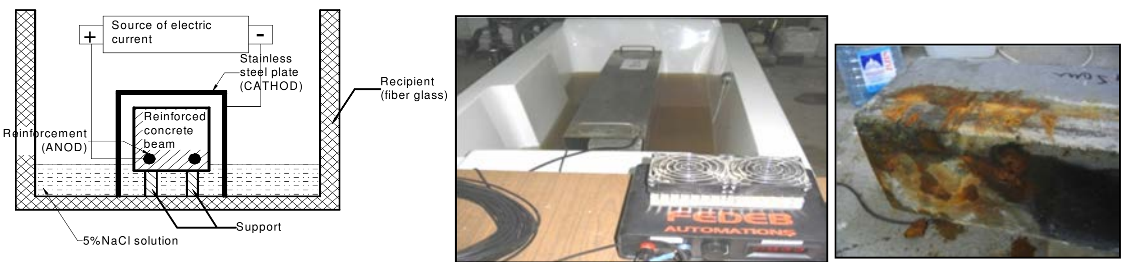

Fig. 2. (a) Schematic view of the corrosion process; (b) Corrosion equipment; (c) Visual aspects of a corroded specimen.

Type b elements exposed to accelerated corrosion already displayed bending cracks along their length, which in fact are the path through which the electrolyte reaches the reinforcement. After only 1 day of exposure, the intensity of the electric current dropped under 0.5 mA/cm^2, indication that the salt bridge of the electrolysis could no longer close the circuit and reach out the steel. In fact, after the exposure, the previous cracks were found to be filled with a light colored material. Although not tested microscopically, the authors believe that a process of self healing took place due to un-hydrated cement particles in low water/binder compositions, as confirmed by other research [2]. Nevertheless, corrosion took, as witnessed by the specific red spots on the surface of the beams (Fig. 2c).

3. Results and discussions

3.1. Main findings

Table 3 presents the experimental values recorded during the loading tests of the elements with concrete cover c=25 mm and c=50 mm. The bending design capacity M_{Rd} was also calculated, using common design equilibrium equations, with high strength concrete coefficients η=0.895 and λ=0.748, based on a characteristic compressive strength f_{ck}=71 MPa, as in relations (3.20) and (3.22), EC2 [15]. Safety coefficients for concrete (γ_c=1.5) and steel (γ_s=1.15) are also taken into consideration. The characteristic crack width $w_{k,EC2}$ was also calculated based on Eurocode 2 relations (7.8), (7.9) and (7.11) [15]. The results of Table 3 are discussed in the following subchapters.

Table 3. Description of the specimens.

	Characteristic	c= 25 mm				c= 50 mm	
		Type (a)		Type (b)		Type (a)	Type (b)
		GDA 1-1	GDA 1-2	GDS 1-1	GDS 1-2	GDA 2-2	GDS 2-2
	M_{Rd} (kNm)	8.320	8.320	8.320	8.320	8.320	8.320
	Mu (kNm)	16.500	14.550	15.375	14.925	13.800	14.100
	M_{Rd}/Mu	0.504	0.572	0.541	0.557	0.603	0.590
$w_{max,exp}$ = 0.1 mm	M/Mu	0.323	0.412	0.585	0.759	0.355	0.521
	$w_{k,EC2}$ (mm)	0.078	0.033	0.058	0.148	0.125	0.159
	Δ (mm)	0.650	0.250	0.466	1.480	1.000	0.900
	M (kNm)	5.325	6.000	9.000	11.325	4.896	7.350
$w_{max,exp}$ = 0.2 mm	M/Mu	0.486	0.613	0.808	0.844	0.457	0.632
	$w_{k,EC2}$ (mm)	0.121	0.143	0.176	0.268	0.166*	0.303
	Δ (mm)	1.150	1.300	1.100	2.800	1.514	1.450
	M (kNm)	8.025	8.925	12.416	12.600	6.300	8.913

3.2. Residual strength

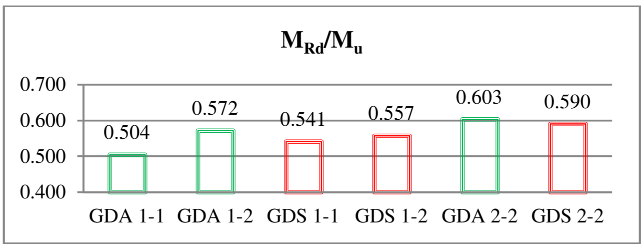

Fig. 3. Residual strength of non-exposed and exposed beams.

Fig. 4 presents the difference between the design resistant bending moment M_{Rd} and ultimate experimental bending moment of both 25 mm and 50 mm concrete cover beams. It can be seen that all the beams, even the corroded ones, display a safety of 39 to 45% up to failure. The corrosion of the reinforcement has little effect on the bending capacity of reinforced high strength concrete beams. The average ultimate bending moment M_u for beams with concrete cover of 25 mm is 2.47% higher for type a beams than for type b beams. However, a less than 5% in experimental testing is usually allowed, due to execution procedures or minor differences in the material properties and not solely to the influence of the corrosion process. This hypothesis is also confirmed on 50 mm concrete cover beams, which present an inverse percentage: type b ultimate moment M_u is higher than type a with 2.13%. The ratio M_{Rd}/M_u renders a safety factor for the beams: for 25 mm concrete cover, an average of 46.2% for non-exposed elements, type a, between the design and the ultimate experimental bending moments and an average of 45.1% for exposed elements, type b. For 50 mm concrete cover, the differences are lower than for 25 mm concrete cover, due to the brittleness of the concrete cover when cracked: 39.7% for type a and 41% for type b.

3.3. Loading step versus crack width

Fig.4 and 5 present the development of the crack width of the 25 mm and 50 mm concrete cover beams under load, as a function of the bending moment. The design crack width $w_{k,EC2}$ is also plotted.

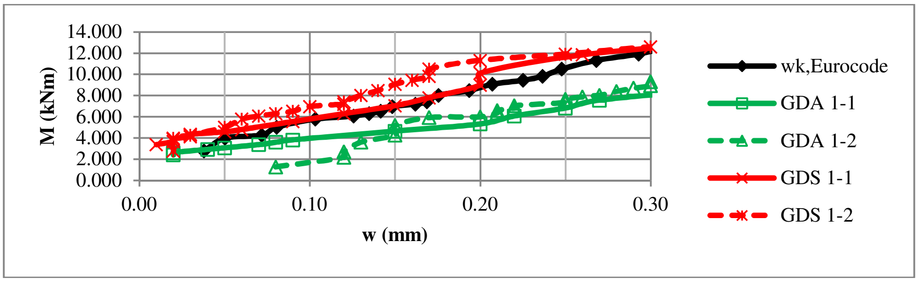

Fig. 4. Bending moment and correspondent experimental and design crack width for beams with c=25mm.

Although the bending capacity for 25 mm concrete cover beams is not significantly influenced by the type of the elements, the maximum crack width recorded during tests is significantly higher for non-exposed, type a beams than for corroded, type b beams. Moreover, the EC2 relations for calculating the crack width carry a conservative approach compared to non-exposed high strength concrete elements, rendering lower crack widths for the same value of the bending moment, as seen from Fig.4.

The same observation can be made for 50 mm concrete cover beams, but in this case, the EC2 relations provide more accurate results, as seen from Fig.5. Therefore, there is an overestimation of the crack width if the relations are applied on high strength concrete specimens and elements affected by corrosion present smaller cracks at the same value of the load, a possible outcome of subsequent hydration of the concrete.

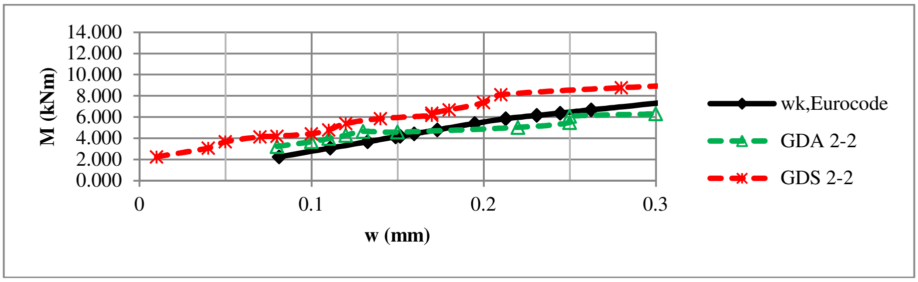

Fig. 5. Bending moment and correspondent experimental crack width for beams with c=50mm.

3.4. Vertical deformations versus crack width

A more detailed analysis is performed in Fig.6 for specific crack widths of 0.1 mm and 0.2 mm, which is also the EC2 imposed limit in aggressive exposure classes. Fig.7 completes the approach, by assessing the vertical deformation at the same crack widths 0.1 mm and 0.2 mm.

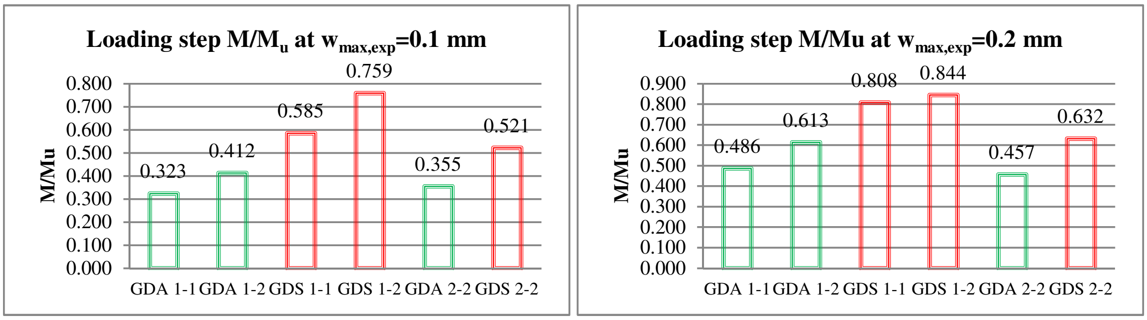

Fig. 6. (a) M/Mu for all beams at $w_{max,exp} = 0.1$ mm; (b) M/Mu for all beams at $w_{max,exp} = 0.2$ mm.

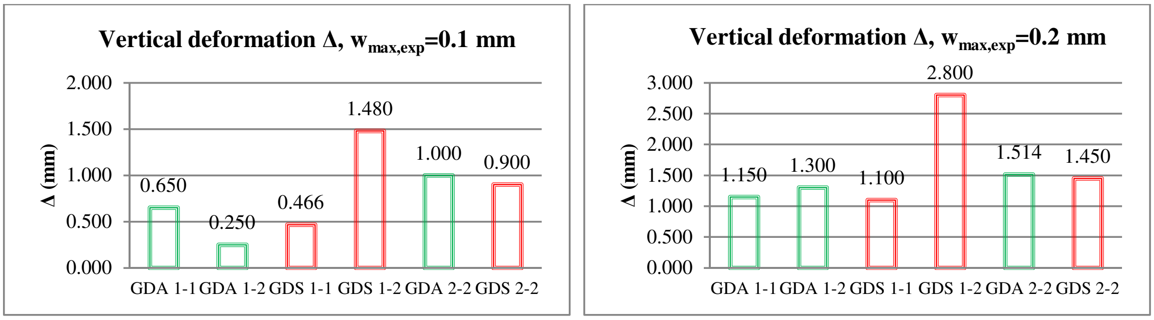

Fig.7. (a) Vertical deformation Δ for all beams at $w_{max,exp} = 0.1$ mm; (b) Vertical deformation Δ for all beams at $w_{max,exp} = 0.2$ mm.

For non-exposed beams with concrete cover of 25 mm, the same value 0.1 mm of the crack width is reached at an average bending moment of 0.368 out of the ultimate bending moment, where an average value of the vertical deformation of 0.45 mm (l_{eff}/2222) was recorded. For corroded beams, the same crack width is reached conveniently later, at an average loading step of 0.672. However, the vertical deformation is doubled, at an average of 0.973 mm ((l_{eff}/1027). Nonetheless, general design rules limit the service limit state at a loading step of 0.4, where the vertical deformation must not rise above l_{eff}/250, in this case 4 mm. For this reason, both non-exposed and corroded beams display a good performance in service, with low crack widths and vertical deformations, with slightly higher deformations in case of the corroded beams. This might be a consequence of the same self healing process, which filled the crack walls with subsequent hydrated cement particles.

The same pattern is observed on 50 mm concrete cover beams: the non-exposed specimens reach a maximum crack of 0.1 mm at a loading stage of 0.355 and a vertical deformation of 1 mm (l_{eff}/1000), whereas the exposed beams develop the 0.1 mm at M/M_u of 0.521 and Δ=0.9 mm (l_{eff}/1111).

A closer situation to the design service period is reached at $w_{max,exp}$=0.2 mm. All loading stages exceed the loading service limit of 0.4, but the vertical deformations do not reach in any case the limit of 4 mm (l_{eff}/250), with the highest deformation, 2.8 mm (l_{eff}/357) reached by one of the corroded beams with 25 mm concrete cover.

4. Conclusions

The current study investigated the influence of the cracking pattern on the structural behavior of reinforced high strength concrete elements exposed to accelerated corrosion, with concrete cover of 25 and 50 mm. Witness specimens were used for comparison. An evaluation was performed on the crack widths of corroded and non-corroded elements along with Eurocode 2 based calculations. These are the main findings of the study:

- The corrosion of the reinforcement has little effect on the bending capacity of reinforced high strength concrete beams and all the beams displayed safety factors between 39% and 46%.
- The maximum crack width recorded during tests is significantly higher for non-exposed, type a beams than for corroded, type b beams, for 25 mm concrete cover. Moreover, the EC2 relations for calculating the crack width carry a conservative approach compared to non-exposed high strength concrete elements, rendering lower crack widths for the same value of the bending moment. Closer measurements to EC2 calculations are obtained on 50 mm concrete cover beams.
- For non-exposed beams, the same value of the studied crack width is reached at a lower loading stage than for corroded beams, but the vertical deformations are larger. Nonetheless, all beams display a good behavior in service with vertical deformations lower than $l_{eff}/250$.

The assessment of the crack widths and the vertical deformations at different loading stages of exposed and non-exposed elements of the present study confers a better understanding and a boost of confidence in using structural high strength concrete. All the beams displayed a good performance in service, with low crack widths and vertical deformations, with slightly higher deformations in case of corroded elements. The conclusions of the study are also an important basis for future research in the area and can be used for improving future design provisions.

Acknowledgements

The authors would like to express their gratitude for Professor Cornelia Măgureanu, director of the research programs CNCSIS 59 and CNCSIS 1552, 'Green concrete. Environmental ecology', for making possible this study.

References

[1] Otieno M, Beushausen H, Alexander M. Towards incorporating the influence of concrete cover cracking on steel corrosion in RC design codes: the concept of performance based crack width limits. Materials and Structures 2012. 45:1805-1816
[2] Marcotte TD, Hansson CM. The influence of silica fume on the corrosion resistance of steel in high performance concrete exposed to simulated sea water. Journal of Material Science 38 2003. 4765-4776.
[3] Raupach M, Dauberschmidt C. Reinforcement corrosion in cracks of high strength concrete-investigations and calculations. 6th International Symposium on Utilization of High Strength/High Performance Concrete. Leipzig 2002. p.1299 - 1312.
[4] Yoon S, Wang K, Weiss W J, Shah SP. Interaction between Loading, Corrosion and Serviceability of Reinforced Concrete, ACI Materials journal 2000, V. 97, No. 6, Title no. 97 – M72.
[5] Solgaard AOS, Michel A, Geiker M, Stang H. Concrete cover cracking due to uniform reinforcement corrosion. Materials and Structures 2013. 46:1781-1799.
[6] Wang Z, Jin X, Jin N, Gu X, Fu C. Cover cracking model in reinforced concrete structures subject to rebar corrosion. Journal of Zhejiang University-Science A (Appl Phys & Eng) 2014. 15(7):496-507.
[7] Al – Harthy AS, Mullard J, Stewart MG. Cracking in concrete due to corrosion of steel reinforcement. CONSEC'07, Vol. 1, p. 383 – 390.
[8] Sanchez AN, Sagues AA, Paredes MA. Reinforced concrete corrosion performance in Florida bridges. RILEM International workshop on performance-based specification and control of concrete durability. p. 533 - 540.
[9] Oh BH, Kim KH, Jang SY, Jang BS. Cracking behavior of reinforced concrete structures induced by steel bar corrosion. CONSEC'07 Tours France. Vol. 1, p. 391 – 398.
[10] Wittmann FH. Crack formation and fracture energy of normal and high strength concrete. Sadhana Vol. 27. Part 4. 2002. p. 413-423.
[11] Mokhtar KA, Loche JM., Friedmann H, Amiri O, Ammar A. Steel Corrosion in Reinforced Concrete, Report no. 2-2. Concrete in marine environment, MEDACHS, Interreg IIIB Atlantic Space – Project no. 197 2006.
[12] Andrade C, Alonso C, Molina FJ. Cover cracking as a function of bar corrosion: Part I - Experimental test. Materials and Structures 1993. 26. 453-464

[13] Phetkaysone A, Ishida T, Sato R. Corrosion cracks and deflection of RC beams subjected to salty water spray under sustained load. Concrete Durability and Service Life Planning – ConcreteLife'09. RILEM Publications SARL. p. 55 – 62.

[14] Azad AK, Ahmad S, Azher SA. Residual Strength of Corrosion – Damaged Reinforced Concrete Beams. ACI Materials Journal 2007. V. 104. No. 1. Title no. 104 – M05.

[15] SR EN 1992-1-1:2004 Eurocod 2: Proiectarea structurilor de beton. Partea 1-1: Reguli generale şi reguli pentru clădiri (RO). Design of concrete structures. Part 1-1. General rules and rules for buildings (EN).